Historical review of prescriptive design rules for robustness after the collapse of Ronan Point

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Abstract

After the collapse of Ronan Point tower building in 1968 there was an unprecedented discussion about the issue of progressive collapse in structural design. In particular, recommendations were published that precast panel structures should include tying elements to hold a structure together after an element loss. The initial investigation into the causes of the collapse and the majority of the subsequent discussion was focused on ensuring precast structures had the same monolithic behaviour as conventional forms. However, the prescriptive recommendations were then applied to all structural forms without amendment to account for the different mechanical behaviour. This paper presents the findings from a novel bibliographic study of historical documents published soon after Ronan Point collapse which influenced the development of relevant design guidelines. The technical information was analysed chronologically to determine the intended purpose of such requirements and the assumptions they were based on. It then traces the development of progressive collapse design requirements to the current Eurocodes to consider if they are being applied as intended. This critical review is timely since robustness considerations in Eurocodes and other international codes are currently being reviewed and general misconceptions regarding existing prescriptive rules have been identified amongst practitioners in the UK and internationally.

Keywords: Progressive Collapse, Design Codes, Robustness, Structural Integrity

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1. Introduction

It is well established that structures must be designed to ensure that they are suitably robust and that they should not fail disproportionately to a damaging event. Therefore modern design codes provide guidelines to help designers with this issue. Many people are aware that the unexpected partial collapse of Ronan Point tower building in 1968 due to a relatively small gas explosion was the starting point for such considerations [1]. However, it is important to understand how the code requirements have changed and developed since then. In particular this matter should be considered carefully at this time as the new generation of Eurocodes 2020 are in development and robustness is in the agenda for EN 1990 and EN 1991-1-7. Additionally, other international codes are being redrafted and consideration taken for their robustness requirements.

Furthermore, the issue of providing suitable protection against disproportionate collapse has been raised by a number of international meetings, for example in 2005 the Joint Committee on Structural Safety and the International Association of Bridge and Structural Engineering, concluded that current codes fail to ensure sufficient structural robustness which is especially concerning for high-risk buildings. This concern led to the development of a European COST project initiative in 2007-2011 (COST Action TU0601: Robustness of Structures). Around a similar time in the US, the ASCE SEI Committee on Disproportionate Collapse Standards and Evidence was formed in order to develop consensus-based design approaches from existing US guidelines. Research on structural robustness has experienced a significant peak of interest as reflected by the number of publications in this topic since 2000, as reported by Adam et al. [2]. The Institution of Structural Engineers (IStructE) have also sought to address this issue with the publication of design guides [3, 4] which include an illustration that shows the locations of the different

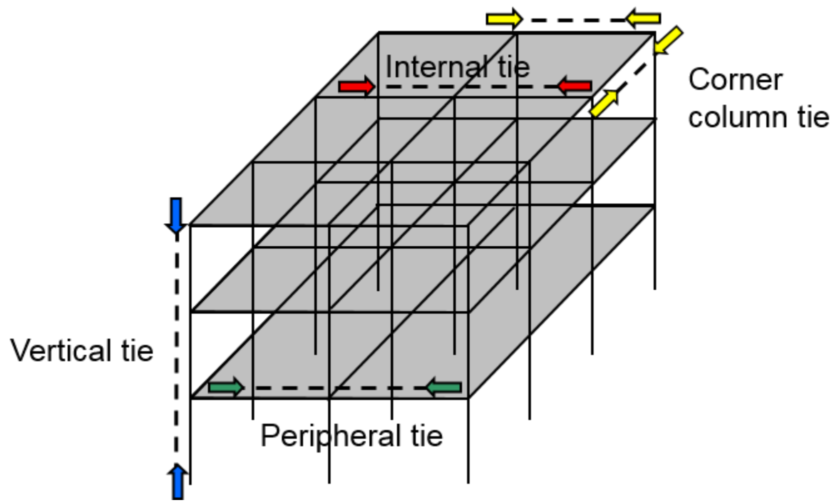


Figure 1: Example of the location of ties in a flat slab structure (IStructE 2010 [3])

types of ties (Figure 1) that are generally recommended in design guidelines. This approach, consisting of a series of prescriptive details rules, is generally adopted by international codes for low risk structures.

With this in mind it is vital that any misconceptions about the use and purpose of the current prescriptive rules for providing tying elements throughout a structure are addressed and understood correctly. Therefore, this paper describes considerations for robustness that were produced in the UK after the collapse of Ronan Point and traces the various iterations during the development of design codes. This allows the intended purpose of the rules to be understood and will help designers and future code writers appreciate the limitations of such approaches. This analysis is timely in the current context of the development of different guidelines for robustness and it is particularly useful to an international audience who might not be familiar with the background of UK building regulations. Some of this background is distributed between different historical documents which are difficult to access and trace.

2. Ronan point collapse and the immediate discussion

Much in depth discussion has been made about the collapse of Ronan Point tower building [1, 5], and this paper does not seek to readdress this analysis or provide additional information on the event itself. Rather the focus is on the immediate discussion and reports that occurred (1968-1972) and their consequences for the structural engineering community. Figure 2 provides a summary of some of the most significant events, covering a time scale from 1905 with an early example of a progressive collapse described in [6] with large social impact but negligible impact on codes and robustness considerations at that time, until 2025 with the expected end of the work on the second generation of Eurocodes.

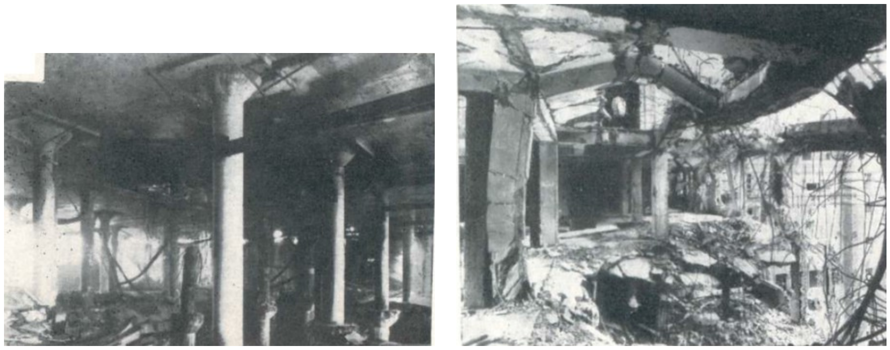
2.1. Initial reports

Shortly after the collapse the UK Ministry of Housing and Local Government released Circular 62/68, 'Flats constructed with pre-cast concrete panels. Appraisal and strengthening of existing high blocks: Design of new blocks' [11], which required an investigation into the susceptibility to progressive collapse for all existing pre-cast load bearing buildings over 6 storeys and a check for fire and wind requirements. It described a method for resisting progressive collapse by providing an alternative load path for the loadings above a removed wall, by utilising "arching, beam, or cantilever action by the provision of suitable reinforcement" [11]. It alternatively specifies that elements could have a local resistance of 5psi (34kN/m^2), described as the maximum likely pressure from an explosion in a block of flats, although the scientific justification of this value was debated [27].

Over the next few months the Institution of Structural Engineers (IStructE) released 5 reports (RP-68-01 to RP-68-05) [12, 13, 15, 28, 29] outlining the Institution's response and opinion to these matters. The first report [12] opened by emphasising their recommendations were focused on prefabricated concrete panels

Date	Event	Relevance
April 1905	Reservoir CYII collapse in Madrid [6]	Early example of progressive collapse
1948	Work by Baker after WWII [7]	First studies on structures subjected to local damage
1954	Work by Walley WWII [8]	
April 1967	CEB bulletin 60 [9]	Recommendations for large panel structures
May 1968	**Ronan Point collapse**	
November 1968	Ronan Point Inquiry Report [10]	
November 1968	Government Circular 62/68 [11]	
December 1968	IStructE reports RP/68/01 & 02 [12, 13]	Guidance for preventing progressive collapse
1970	Addendum to CP116 [14]	
1971	IStructE report on the fifth amendment RP/68/05 [15]	
1972	Code of Practice 110 [16]	First code to consider progressive collapse
1985	British Standards for concrete and steel [17, 18]	
1992	Building Regulations 1991. Approved Document A [19]	Includes disproportionate collapse requirements
1995	AP Murrah Federal Building collapse	
1998	ASCE 7-98 [20]	Comments on structural integrity
2001	World Trade Centre collapse	
2002	Eurocode 1990: Basis of structural design [21]	
2003	GSA guidelines [22]	Specific document for progressive collapse of US federal buildings
2009	IBC 2009 [23] and UFC 4-023-03 [24]	
2013	GSA update [25]	
2016	Extension to ASCE 7-98 [26]	
2015-2025	Work on second generation of Eurocodes	WG6 review clauses of Robustness

Figure 2: Time line of major events and documents



(a) Local damage leading to the loss of one column and slab in a flat slab building

(b) Local damage leading to frame damage and partial loss in a frame building

Figure 3: Arrest of progressive collapse and damage of monolithic RC buildings after internal explosions during WWII (Baker et al. [7], reproduced with permission of ICE Publishing)

used in residential buildings and added that for conventional construction (i.e. not precast concrete) "...experience has shown that the structures are capable of safely sustaining abnormal condition of loading and remaining stable after the removal of primary structural members." Some of this experience was based on field observations after World War Two that had demonstrated that monolithic RC structures showed good robustness to local damage [7, 8, 30]. Figure 3 shows some concrete structures that experienced local damage due to explosives but maintained integrity, it should be noted that Baker [7] does talk about the role of structural ties in his description of building failures.

In commenting on the application of the government's circular the IStructE's report adds the comment that if the wall panels are not connected to the floor slabs then "... adjacent floor panels may act as a catenary over a twin span of a floor" [13], although this is limited to a sag of half the storey height, and it was acknowledged this is dependent on the connections.

An investigation into the Ronan Point collapse event was commissioned and its findings presented in the 'Report of the Inquiry into the Collapse of Flats at Ronan

Point, Canning Town' published in the same year [10]. This report was thorough in its consideration of the failures involved ranging from flawed structural design to poor workmanship and out of date codes of practice. However, it is clear that its conclusions are intended to be very narrow in application. This arises from the fact that the report limited itself to precast system buildings, and indeed only those of the construction used for Ronan Point. The main recommendations from the report are that consideration should be given to the use of town gas in high rise buildings, existing, tall 'system-built blocks' should be appraised and that codes of practice should be brought up to date. This last point focused on wind loading on high buildings and for large concrete panel construction.

There is however a simply worded recommendation presented that "the Building Regulations should include provisions dealing with progressive collapse." The report commented that the existing steel or reinforced concrete frame tall buildings "...are not liable to progressive collapse and accordingly nobody turned their minds to this specific question" [10], and so, according to report, it was only due to the use of new construction techniques that resulted in this becoming an issue.

The report also identified that large concrete panel system buildings requires "continuity at the joints of a kind strong and tough enough to stand both the initial shock of local damage and the abnormal and, in detail, unforeseeable loads they may subsequently have to bear" [10]. In fact, the aim was to achieve a non-brittle monolithic structure. The authors state "Reinforced concrete buildings constructed of in-situ concrete have most of the properties required" [10].

The report authors stress the fact that it does not consider it appropriate that they should attempt to deal in detail with the measures needed to strengthen the joints. However, they comment that a "generous and general distribution" of mild steel between panels would have improved the design.

In summary, any comments from the Ronan Point inquiry are very narrow in terms of application, dealing exclusively with large precast concrete panels. The recommendations presented are mainly concerned with the out of date Code of Practice for wind loading and the use of town gas in tall structures, as well as the introduction of a code of practice for panel buildings. It is also understood that the authors believed that progressive collapse is not a major concern for frame structures or in-situ concrete. Finally, while broadly stating the requirement for a structure to be tied together the report makes no detailed comment on how this should be achieved, or the design loads that would be appropriate. Additionally, common later terms such as *cantilever, catenary, ties, robustness* or *sudden column loss* are not used at all.

2.2. Early discussions on code amendments

A Short and J R Miles' 1969 [31] paper on the new draft requirements for large panel structures [14] describe the purpose of tying elements as to "ensure the integrity of the construction, to prevent structures from falling apart." This is contrasted to monolithic structures where "these ties forces are generally present owing to the inherent nature of the construction" [31].

Short and Miles also expand further on the role of ties in different locations (see Figure 1 for examples), stating the reinforcement along the peripheral will strengthen the corner and "produces the necessary bridging reinforcement to take care of the danger of progressive collapse caused by the elimination of an external wall" [31]. Whereas internal and peripheral ties "are intended to impart a quasi-monolithic character to the floor" [31] which could allow membranes to form. Within their conclusion they state that due to the lack of reliable scientific evidence the "requirements were based on the intuitive judgement of experienced structural engineers backed by a design study."

Finally, although they recognised that the addendum was only intended for large-panel structures they argued that the principles should be "considered for other types of construction that are sensitive to exceptional and unforeseen loadings" [31]. This went beyond the report into the collapse of Ronan point which was limited to precast construction, and recommended only investigation and new regulation for large panel structures. However, the government decided the fifth amendment to the building regulations should to apply to all structural forms. This was a matter of some controversy and was among the issues debated on this subject in the UK House of Commons, as Nicholas Ridley MP (himself a qualified civil engineer) stated, "Instead of accepting the recommendations of the report, [the government] launched out on their own with a series of building regulations which are far too technical and far too expensive and which will do great harm to the advancement of building techniques" [27].

However, the Minister of State for Housing at the time, Mr Reginald Freeson MP, stated that "The amendment proposals are primarily aimed at protection against the effects of damage, however it is caused, or against progressive collapse experience, however it might be caused, in the future" [27] and then explained why the new provisions were applied broader than just precast structures. He viewed that the "new mandatory requirement should not be selective in its application and so not be thought to penalise a particular form of construction" [27]. He also argued that some framed and brick buildings were still susceptible to progressive collapse, but the new requirements would not be unreasonable for structures that were not susceptible and that "the new requirements have been deliberately drawn up in fairly functional terms...to facilitate their application to varying forms of construction and to permit of the flexibility of design" [27].

By the time the code of practice for the structural use of precast concrete, CP116 [14], was updated to include the Ronan Point Inquiry Report's recommendations, some aspects took the lead from a previously published CEB recommendations for large panel precast structures [9]. This document suggested that the panels should be tied together to maintain continuity. However, the purpose of those ties was to resist horizontal loadings (e.g. wind, seismic, eccentricities) and forces due to differential settlement and no considerations was given to element loss. Similarly, as result of Ronan Point and the subsequent updating of regulations, Fintel and Schultz conducted investigations during the 1970's into the integrity of large panel buildings in America [32]. Their recommendations of including ties between elements to maintain integrity and bridge over potential damage were developed as a basis for the minimum tie force requirements still in use in the ACI Standard 318 [33].

After building regulations and some codes of practice were updated to include the Ronan Point Inquiry Report's recommendations, the Institution of Structural Engineers published the report "Stability of modern buildings" [34] which notes on the topic of providing alternative load paths that "...there is no fully detailed design procedure by which to implement this philosophy. There is, however, plenty of practical experience to demonstrate that a damaged structure will remain stable in favourable circumstances after an incident that has severely damaged and possibly removed a section or component of the framework" [34]. This report also viewed that the formation of a catenary or membrane could maintain stability after a damaging event, as long as the floors and columns are tied together. For box-frame structures they also state that for an effective tie "The magnitude is obviously related to the span of the floor and beam components and should be capable of sustaining practical catenary effects" [34].

Research into the issue of progressive collapse continued into the 1970's and 80's with particular interest in concrete structures and catenary action [32, 35–38]. However, there was no major changes to design approaches as a result of these later studies, except for some rules introduced for detailing specific structural elements.

The original report into the Ronan Point collapse made clear that its recommendations were intended only for precast, indeed it states that RC frames are unlikely to undergo progressive collapse. Its proposed method of preventing progressive collapse was to attempt to give precast structures the same monolithic behaviour that occurs naturally with in-situ construction. Although it is now known that RC frames and slab structures may still be susceptible to progressive collapse, (for example the Skyline Plaza in 1973 and Sampoong Department Store in 1995 [39]) this is due to their different mechanical behaviour (e.g. brittle connection failures, long spans, less optimal construction control compared to prefabricated structures) rather than their loss of continuity.

3. Review of code tying force requirements

3.1. CEB bulletin 60

In 1967 CEB released a report entitled ‘International recommendations for the design and construction of large-panel structures’ [9]. This document included some broad guidance for ensuring the continuity of precast panels such as “Within the thickness of each floor, or close to the floor, mechanically continuous steel “ties” should be provided in both directions.” Both peripheral and internal ties (or chains to use the original French term) were required, although these are specified between panels and not flooring elements. The purpose of these are described as:

- resist the forces acting on the external panels (in their load-bearing role) owing to inaccuracies of installation;
- resist the horizontal reactions directly exerted by the panels (wind and earthquake);
- resist the tensile forces developed in the floors performing their wind-bracing function;

- resist where necessary the horizontal components of the diagonal forces in the wall panels subjected to tensile load by the effect of lateral forces, and transmit the horizontal floor reactions to the windward edges of the wind-bracing cantilevers;
- resist the tensile forces developed in the walls if differences of level develop in the supports

From a review of these clauses it is clear that although most of the terminology used (e.g. peripheral and internal ties, continuity, cantilevers etc.) is similar to later progressive collapse design guidelines, these requirements were not intended to address the sorts of extreme scenarios that occur after a sudden column loss. Although, it has been noted that if Ronan Point had been designed and constructed to comply with CEB bulletin 60 the consequences of the event would have been significantly lower [40].

3.2. CP 110 : Part 1 : 1972

One of the earliest national codes to include consideration for progressive collapse was Code of Practice 110 in 1972 [16]. Here the requirement is stated that, "The layout of the structure on plan, and the interaction between the structural members, should be such as to ensure a robust and stable design...there should be reasonable probability that it will not collapse catastrophically under the effect of misuse or accident." It is also clarified that the structure is not expected to resist extreme loads of events, but not fail disproportionately. To meet this requirement ties are recommended to maintain the structures robustness. Initially the purpose of ties in buildings was stated in the additional clause that "...the ties should be so placed as to provide the best assistance in resisting by *cantilever*, *catenary* or *other actions* the results of extreme damage by accidental causes." Emphasis added to highlight the mechanisms by which the ties are utilised for, and therefore the expected state of the structure for which they are relevant.

The required tensile capacity of these ties depends on their location within the structure, however all are based on a function of F_t, in kN. This value is a function of the number of stories, with an imposed minimum. From this it can be seen that it is assumed that a taller structure requires higher tensile capacity to maintain integrity, although this is capped at 10 floors. Therefore F_t has a range of between 24 and 60 kN.

$$F_t = \min \begin{cases} (20 + 4n_s) \\ 60 \end{cases} \text{kN} \tag{1}$$

Peripheral ties must resist F_t kN within 1.2 m of the edge of the building. External columns and wall must also be capable of resisting a force which increases with the floor to ceiling hight, l_o. This value varies between 0.8 and 2 times F_t for floor to ceiling heights of 2 to 5 meters, and is capped at $2F_t$. Additionally this value must be greater than 3% of the ultimate design vertical load. $F_{tie,col}$ is given in Equation 2. Note the symbol $F_{tie,col}$ is not used within CP 110.

$$F_{tie,col} = \max \begin{cases} \min \begin{cases} 2F_t \\ (\frac{l_o}{2.5})F_t \end{cases} \\ 3\% \text{ of the ultimate vertical load} \end{cases} \text{kN} \tag{2}$$

Internal ties must resist at least F_t kN *per meter width*, with larger values required for longer spans or larger characteristic loading. Assuming the floor to ceiling height is less than 4m and the span is under 10 m, then for typical loading levels the tie force is under $2.67F_t$ kN/m.

$$F_{tie,int} = \max \begin{cases} F_t \\ \frac{F_t(g_k+q_k)}{7.5}\frac{l}{5} \end{cases} \text{kN/m} \tag{3}$$

g_k and q_k are the characteristic dead and imposed UDLs and l is the span length, limited to 5 times the clear story height.

Finally vertical ties, acting across all levels should be provided. The area of reinforcement specified should be at least the minimum requirement for normal conditions.

Based on these requirements a number of underlining assumptions can be seen. Firstly that if a structure loses its primary load path, it is assumed secondary mechanisms could be utilised and the structure can develop the required tie forces, without additional checks. The tensile force within these ties can usually be determined based on the overall structural geometry, i.e. number of stories, storey height, span lengths. Only internal or vertical ties require consideration of the direct loading on that floor. Additionally, as the consequences of failure of a taller structures may be more severe, the secondary mechanisms must resist higher forces to reduce the chance of progressive collapse. Finally, by providing vertical ties, the probability of a column failure is reduced and there is potential for load sharing between floors, although this mechanism is not explicitly checked.

3.3. BS 8110 : Part 1 : 1985 and 1997

The British Standard, BS 8110 [17], for concrete was introduced in 1985. Clause 3.12.3 of this document stated that "The necessary interaction between elements is obtained by tying the structure together using the following types of tie..." [17], and described the use of horizontal and vertical ties. It is noteworthy that reference to the mechanisms by which ties operate (e.g. cantilever, catenary) have been removed and replaced with a general statement regarding tying the structure, however the section continues in an almost identical wording and requirement to CP 110, indicating they are based on the same assumption. The requirements for vertical ties is expanded on in this standard and introduces the additional requirement that the ties must able to carry in tension the vertical design loading on that column from one floor. The update to BS 8110 in 1997 [41] made no change to the requirements.

3.4. BS 5950 : Part 1 : 1985, 1990, 2000

For steel structures, the original BS 5950 in 1985 states in the introduction that "The structure should behave as one three-dimensional entity. The layout of its constituent parts, such as foundations, steelwork, connections and other structural components should constitute a robust and stable structure under normal loading to ensure that in the event of misuse or accident, damage will not be disproportionate to the cause" [18]. It also highlights that minor incidental loads should not jeopardise the safety of other parts.

The requirements of the tie include the following clauses: "All ties and their end connections should be of a standard of robustness commensurate with the structure of which they form a part and should be capable of carrying a factored tensile load of not less than 75kN at floors or 40kN at roof level." This is clarified later with the requirement that the ties should be, where practical, arranged in continuous lines and at two directions. The required tensile strength of these ties, and their connections, is given in Equations 4 and 5, for internal and periphery ties respectively.

$$T_i = 0.5 w_f s_t L_a \tag{4}$$

$$T_p = 0.25 w_f s_t L_a \tag{5}$$

where w_f is the factored loading, per area, on the floor. s_t is the spacing between ties and L_a is the length of the tie under investigation.

3.5. Approved Document A

Within the UK, the publication of Approved Document A [19] provided additional guidance for designing structures against disproportionate collapse. This document is equivalent in status to a code of practice, i.e. that compliance with the guidance does not absolve the designer of his legal responsibilities. The legal requirement is that contained in the Building Regulations 2000 (as amended), which state that "the building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause." The document is based on the principle of categorising buildings into consequence classes based on their size and use and describes the provision of ties and the conducting of alternate load path analysis. The latter approach is necessary if more than 15% or $70m^2$ of floor area would collapse after an element removal. The $70m^2$ is equivalent to two perimeter bays on a 6m structural grid. However, Eurocode 1991-1-7 [42] increases this limit to $100m^2$ in recognition of increasing structural spans (equivalent to two perimeter bays on a 7.5m grid) and later revisions of Approved Document A incorporate the same change. If such an alternate load path cannot be achieved, the element is defined as 'key' and must be able to withstand a notional accidental load of $34kN/m^2$.

The 1992 edition also made an attempt to allow designers to avoid or reduce the hazards to which the building may be exposed (for example removing potentially explosive sources or keeping vehicles from approaching a building), as an alternative to any of the structural measures described. Whilst a conceptually valid approach, it is unlikely this would be sufficient to justify the omission of any tying or robustness requirements from a design. However it may be a consideration when dealing with existing buildings, particularly those designed pre-Ronan Point. This approach, as an alternative, has been deleted from subsequent editions of the approved document.

In 2004 the document was revised to extend the previous requirements, mandating horizontal tying for buildings below five stories for the first time (except single-occupancy residential dwellings). Above five stories, the previously-existing general requirements continued to apply (horizontal and vertical tying, alternate load path analysis and key element design). Basements became excluded from the storey count, provided they satisfy Class 2B, potentially to reduce the requirements for horizontal tying. A significant difference with the 2004 edition was the subtle change to the layout and punctuation that resulted in horizontal tying no longer being required with notional element removal. However, in previous editions of Approved Document A, the intent is quite clear: horizontal ties are to be provided regardless of whether vertical tying or alternate load path analysis is chosen.

This edition also includes the option of "effective anchorage of suspended slabs to walls" given as an alternative to horizontal tying. Although similar to the provision of horizontal ties, reference to masonry code reveals that it is intended to describe deemed-to-satisfy details in masonry construction of joist hangers for timber floor joists or built-in ends of precast concrete slabs which satisfy the Class 2A requirement without tying reinforcement. The Code gives no detail about the resilience of the details proposed, but as they are only valid for Class 2A buildings, it can be inferred that they provide less resilience than full horizontal tying. Additionally, note that as prior to 2004, horizontal tying did not apply in buildings less than 5 stories, this potentially allowed a reduced requirement for the masonry and timber industries for this building class.

Finally, in 2004 the Class 3 category, was introduced covering the highest risk buildings, for which systematic risk assessment is required. Intended for buildings exceeding 15 stories or 5000m^2 per storey, those containing hazardous substances or processes, stadia and grandstands, or buildings into which the public are admitted in significant numbers. The IStructE (practical guide to robustness) also recommend that certain buildings merit treatment as Class 3 buildings because of their value, vulnerability or the consequences of their failure. [4, 43].

3.6. Eurocode

With the introduction of the Eurocodes, a number of changes were made regarding robustness considerations in design. However, the majority of these refer to a reorganisation of design requirements rather than a substantial change in methodologies.

3.6.1. EN 1990

Eurocode EN 1990 [21] states as a basic requirement that "A structure shall be designed and executed in such a way that it will be not be damaged by events such as: explosion, impact, and the consequences of human errors, to an extent disproportionate to the original cause." On inspection, this is just a restatement of the requirement that existed in CP 110. Under the list of options for avoiding or limiting the potential damage within EC0's basic requirements, three are applicable for this study. These are "selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage; avoiding as far as possible structural systems that can collapse without warning; and tying the structure together" [21].

3.6.2. *EN 1991-1-7*

EN 1991-1-7 [42] provides in the information for tie strengths, and for the first time, explicitly separates frame and load-bearing wall construction, although its requirements cover all structural material forms.

For framed structures, the ties must be able to carry the required tensile load, according to Equations 6 and 7 for internal and perimeter ties respectively which given as a function of the loading and the span length. These equations and methodology are similar to the previous British Standard for steel structures (BS 5950 [18]), see Section 3.4. The separate requirement for external columns or walls to be tied in has been moved to the different material sections.

$$T_i = 0.8(g_k + \psi q_k)sL \text{ or 75kN, whichever is the greater.} \tag{6}$$

$$T_p = 0.4(g_k + \psi q_k)sL \text{ or 75kN, whichever is the greater.} \tag{7}$$

For load bearing wall cases, a structure in Class 2b risk group, typically buildings between 4 and 15 storeys, requires horizontal ties in the floors. The value of F_t is similar in practice for internal and peripheral ties as previous codes, although the load used in the equivalent equation is that for the accidental situation rather than the ultimate limit state.

3.6.3. *EN 1992-1*

For concrete structures EN 1992-1 [44] provides further instruction. Firstly, the purpose of tying systems is stated clearly: "Structures that are not designed to withstand accidental actions shall have a suitable tying system, to prevent progressive collapse by providing alternative load paths after local damage" [44]. Then follows the list of four types of ties, that have been mentioned in previous codes above. i.e. peripheral, internal, horizontal column/wall ties and vertical ties. Section 9.10.2.2

in EN 1992-1, provides recommendations for these tie forces, however, the UK National Annex differs in calculating these values. For peripheral ties, EC states $F_{tie,per}$ from Equation 8.

$$F_{tie,per} = l_i \cdot q_1 \leq Q_2 \tag{8}$$

where l_i is the length of the end-span and EC recommends $q_1 = 10kN/m$ and $Q_2 = 70kN$. However, the UK decision in their National Annex is to maintain the previous equations from the CP110 and BS 8110, with $Q_2 = 60kN$ and defines q_1 with Equation 9.

$$q_1 = (20 + 4n_o)l_i \tag{9}$$

This can be simplified down to the system used for CP 110 and BS 8110, i.e. $F_{tie,per} = F_t$.

For internal ties, EC simply recommends a value of 20 kN/m while the UK again sticks with existing equation. For most applications the UK requirements of tying forces are higher than the EC suggested values. For floors without screeds and where ties are grouped at beam lines then EC suggests Equation 10 while the UK uses the previous internal tie equation.

$$F_{tie} = q_3 \cdot (l_1 + l_2)/2 \leq q_4 \tag{10}$$

with $q_3 = 20kN/m$ and $q_4 = 70kN$. l_1 and l_2 are the span lengths either side of the beam.

Edge columns and walls should be tied horizontally into the structure. EC recommends $F_{tie,fac} = 20kN/m$ and $F_{tie,col} = 150kN$. Again, the UK decision is to maintain the historical approach, with the same value for columns and walls, although the units differ.

3.7. US provisions

Although this paper is focused on the direct development of UK progressive collapse guidelines as a result of the Ronan Point collapse, other counties have naturally developed their own recommendations [2].]. Many of these international codes show the evolution of some of the early concepts introduced soon after Ronan Point such as the concept of robustness itself or the fundamentals behind the notional element removal approach. This evolution was particularly relevant in the United States of America, especially after events such as the Alfred P. Murrah Federal Building collapse in Oklahoma in 1995 and the World Trade Center in New York in 2001 leading to a number of well established codes such as ACI 318 [33], ASCE 7-16 [26], IBC 2009 [23], UFC 4-023-03 [24] and GSA [25].

ASCE 7-16 [26] describes a requirement of general structural integrity with similar language to Eurocode 1990 stating that a structure must be able "to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage". It lists continuity, redundancy and ductility as methods to achieve this. UFC 4-023-03 Design of Buildings to Resist Progressive Collapse [24] describes a tie force method very similar to European approaches stating that "the building is mechanically tied together, enhancing continuity, ductility, and development of alternate load paths" by use of vertical, longitudinal, transverse, and peripheral ties. Significantly, the existing structural members can only be used to provide the tie force if they are cable of undergoing a 0.20rad (11°) rotation. The latest General Services Administration guidelines (GSA) [25] move away from tie force requirement to focus on alternative load path methods requiring full analysis of the structure after the loss of a load bearing element. Stevens et al. [45] provides an overview of Department of Defense (DoD) requirements for high risk structures, typically government or military.

The majority of approaches mentioned above remain threat independent, although in cases where a known threat exists more detailed design is provided for. More usual designs include tie force checks for low risk locations and alternative load path analysis in more critical scenarios. A general classification of design methods against disproportionate collapse was put forward by the ASCE SEI Sub-committee on Terminology and Procedures [46], dividing methods into non-structural and structural. The non-structural methods focuses on event control whereas the structural focuses on the collapse resistance (robustness and vulnerability), e.g. alternative load path method, segmentation, protection, increased local resistance (key element design). A similar design strategy is proposed in Eurocodes [42] for accidental design situations although in this case the same methods are classified according to whether the accidental action is identified or the method focuses on limiting the extent of localised failure. In both classifications, it is understood that the tying approach would contribute towards limiting the extent of the localised failure and provide integrity.

4. Discussion of code development at an international context

From considering all the development of the requirements, culminating in the current Eurocode guidelines, a number of points can be made. Firstly, it is clear that although the collapse of Ronan Point, which triggered the introduction of robustness considerations in design guidelines, only occurred due to a combination of the nature of the structural form used and deficiencies in design, the solutions were equally applied to all forms of construction. This was not due to technical considerations, but to make the new codes of practice more general in their application and to prevent the precast industry becoming at an economic disadvantage. Related to this, the initial purpose of the ties was to help precast structures behave as monolithic insitu concrete ones, with the view that these forms were able to resist disproportionate collapse. It is now known that such structures may fail despite continuity, therefore, the same methodology may not be appropriate for all structural forms.

The majority of the knowledge about collapse of structures in 1968 was based on observations during World War Two and specialised knowledge for the precast industry. There is very little experimental and theoretical research from the time period to back up the prescriptive rules proposed to prevent progressive collapse, and even less to provide evidence for the tying forces and values prescribed.

It appears the early codes intended tie requirements to be a broad, prescriptive method of ensuring that a structure does not fail disproportionately to a local damaging event. However, the actual method by which this is accomplished, is general at best and vague at worst with different mechanisms described at different times (e.g. catenary, membrane, beam action, arching, cantilever). In addition, due to the prescriptive nature of the tying requirements it is not possible to quantify their effectiveness or what minimum level of robustness they provide. However, despite its limitations, these and similar rules are used in many codes in addition to Eurocode (e.g. ASCE 7-16 [26], IBC 2009 [23], UFC 4-023-03 [24] and also the Chinese code CECS 392:2014 [47] and Canadian NBCC 1995 [48]). However, such an approach is not universal as other international codes, such as GSA 2013 [25], NYC BC 2014 [49] (New York) and NCC 2016 [50] (Australia), exclude this method. See Adam et al. for more information [2].

The traditional prescriptive based codes do not give any consideration to the fact that a structure may not be able to utilise these ties without deforming to an

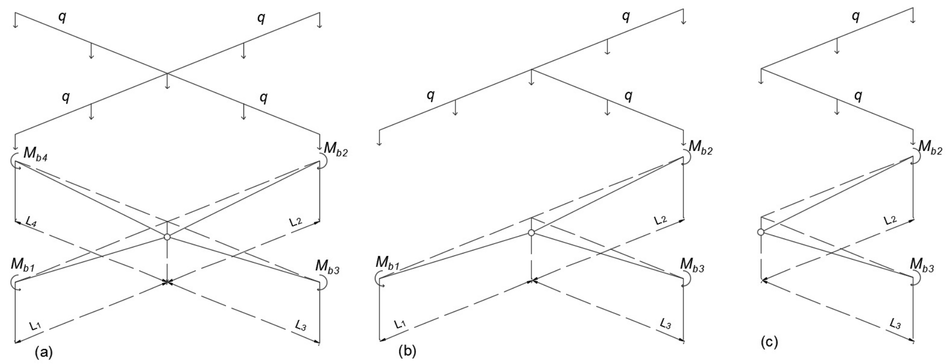


Figure 4: Tie mechanism for a) internal, b) border and c) corner column failure locations from Chinese code CECS 392:2014 [47]

extent which would cause brittle failure. For example, in concrete flat slab construction it is known that punching around the column will occur before the slab can reach the catenary deformations needed to resist gravity loads by means of tensile membrane action alone [51, 52]. However some codes (e.g UFC 4-023-03 in the US and also the Chinese code) are starting to introduce rotation limits for the ties and linking the tying force to physical concepts [53], which is a step forward according to the authors. See Figure 4 from Chinese code CECS 392 that includes ductility and consideration of the beam end moments whilst utilising the tie.

5. Conclusions

After the progressive collapse of Ronan Point tower building in 1968 there was an unprecedented discussion about how buildings were designed and constructed and whether existing or future structures were vulnerable to such events. The investigating report into the incident concerned itself exclusively with large panel precast construction used for Ronan Point and along with concerns with the use of gas in tall buildings it recommended that design codes be brought up to date, especially for wind loading. Soon after the accident the British Government released amendments to the building regulations requiring considering of progressive collapse, and applied these specifications to all structural forms. From the bibliographic study carried out in this work it was concluded that:

1) The new requirements for ties are based on a methodology proposed in CEB bulletin 60 [9], this document predated the collapse of Ronan Point and was concerned with maintaining integrity across precast joints due to minor eccentricities and so they were not intended for use with other structural forms or for dealing with extreme events such as element removal. The literature shows different interpretations of the purpose of the tying approach, including preventing local failure and maintaining integrity after member loss.

2) While the original investigators were no doubt aware of the limitations and the focus of their recommendations, the current form of the guidelines, developed through the British Standards and now the Eurocodes, is not directly based on their scenarios. Although the provisions were based on the best information and experience available at that time, further experimental testing and solid research is needed to support these prescriptive rules.

3) There are significant mechanical differences in the behaviour of large precast panels and modern construction methods and therefore the same approach may not be appropriate. This causes a problem for drafting general rules for structures (such as for the Eurocodes) as the vulnerability and behaviour could be very different for different structural forms.

4) Although the original intention of the ties was to prevent brittle failure, this is not explicitly checked and therefore it is not certain that the tie, or secondary

mechanism can form without further failures occurring. This issue is being addressed in some codes and is an important area of future development.

5) Future revisions for international design codes such the Eurocodes should be orientated towards improving confidence in prescriptive rules and their range of applicability. Such rules should be reviewed based on the variability of the risk of progressive collapse, form of construction, hazards identified and robustness considerations during the design, construction and operation phases.

There have been very few cases of complete progressive collapse in the last few decades and so, whilst useful for practical cases of low risk, it is not possible to quantify the efficiency of implementing ties in building structures. In many cases, designers are unaware of the intended purpose and assumptions behind these guidelines. In some cases applying these guidelines may lead to the false sense of security and in some other cases designers might lean towards alternative load path approaches which are quantitative. With this in mind it is therefore recommended that designers using the current tie guidelines should also take into account the required mechanisms for tie to be utilised after an extreme event. In particular joint ductility for large deformations and the prevention of brittle failures (e.g. shear failures) is vital for such an approach to be effective.

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